Seismic Performance and Design of Bolted Steel Moment-Resisting Frames

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ABSTRACT

Bolted moment-resisting frames, and their predecessors riveted frames, were used for many decades prior to the use of welded frames in steel structures. Riveted and bolted steel structures have performed well during past earthquakes from the 1906 San Francisco earthquake to the infamous 1994 Northridge, Los Angeles earthquake. This paper presents information on past performance of bolted steel moment-resisting frames, a summary of shaking table tests, comparative studies of bolted and welded moment frames and a summary of results of cyclic tests of bolted top-and-bottom flange plate moment connections.

The paper also presents the concept of performance-based design of steel connections using a failure mode hierarchy. In this concept, all failure modes of the connection are identified and then an order of desirability is assigned to each failure mode based on its ductility. The more ductile the failure mode is the higher its place in the hierarchy. Then, for each failure mode, design equations are developed. These equations ensure that, the more ductile failure modes, such as yielding of steel, will occur first and protect the connection from experiencing the more brittle and undesirable failure modes, such as fracture of welds, bolts or net sections. As an illustration of the procedure, this paper presents application of a proposed "hierarchical" approach to failure modes of bolted top-and-bottom flange plate moment connections and provides corresponding design equations.

BACKGROUND

With the development and application of high-strength bolts in construction of steel structures, in the1950s, high strength bolts gradually replaced rivets. During this period, particularly after WWII, structural welding technology was being developed rapidly and economical welded steel structures started to be erected in seismic areas. As a result, the current stock of steel structures in seismic areas includes a variety of riveted, bolted and welded structures.

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In recent years, probably due to the ease of fabrication and economical reasons, most of the steel moment frames used in highly seismic areas such as California have had field-welded moment connections. However, welded steel moment frames are only one of the many possibilities. In the aftermath of the Northridge earthquake and its associated damage to field-welded moment-resisting frames, it appears that bolted moment frames are again becoming an economical and reliable option. Structural engineers are designing and using these bolted systems successfully in their buildings. The main purpose of this paper is to present information on the seismic behavior and design of steel moment frames with *bolted* or *bolted/welded* connections.

TYPES OF STEEL MOMENT-RESISTING FRAMES

Steel moment-resisting frames can be divided into several categories based on

1. configuration of the moment-resisting frame;
2. type of connectors used, i.e. rivets, bolts or welds;
3. ductility of the connections;
4. relative rotational stiffness of the connections and members;
5. relative moment capacity of the connections and members.

These categories are discussed in the following sections.

Types of Moment-Resisting Frames Based on the Frame Configuration

Common categories of moment-resisting frames based on configuration of the frame are

a) space, perimeter and planar moment-resisting frames;
b) column-tree moment-resisting frames;
c) moment-resisting frames with truss girders.

Space, Perimeter, and Planar Moment Frames

A typical *space* moment-resisting frame is a three-directional structural system composed of columns, girders and connections to resist the applied load primarily by flexural stiffness and strength and dissipate energy by ductility of its members and connections, with or without the aid of the horizontal diaphragms or floor bracing systems (ICBO, 1997). In welded space frames, usually all girder-to-column connections are designed and fabricated to act more or less as rigid moment connections.

In a *perimeter* moment frame system only the exterior frames are moment frames acting as a rigid box to resist the lateral load of the entire building by flexural stiffness and strength and dissipate seismic energy by ductility of the members and connections. In this system, the interior columns and girders are not assumed in design to be part of the lateral load resisting system and are connected by shear (simple) connections. These shear connections are assumed to carry only gravity shear load with little or no contribution to lateral load resistance. Recent studies (Liu and Astaneh-Asl, 1999) clearly indicate that there is considerable amount of moment capacity and rotational stiffness in today's shear connections that are currently ignored in design. Such moment capacities are part of the structural system and contribute to the strength and stability of moment-resisting frames (Astaneh-Asl, Shen and D'Amore, 1998).

During an earthquake, not only the gravity columns, girders and their shear connections participate in the lateral load resisting system to some extent, but also the floor diaphragms and some non-structural elements provide stiffness, strength and damping. This is due to the fact that during earthquakes, the entire building is shaken and all elements of the building, including non-structural elements, undergo deformations and rotations. This issue has been recognized by the codes. For example, the Uniform Building Code requires that the shear connections of leaning columns be designed to accommodate deformations (rotations) imposed on them by the lateral displacement of the moment frames.

By using the steel perimeter moment frames instead of the space moment frames, the number of rigid moment connections is reduced, in many cases to less than one half of the number of rigid connections in the comparable space frame. Since the cost of moment connections is much higher than the shear connections, by using a perimeter moment frame, some cost saving over space frames is achieved. However, in doing so, the redundancy of the lateral load resisting system is also reduced. One of the advantages of the perimeter moment frame systems is that the girder spans of the perimeter frames can be quite small. The close spacing of the columns in perimeter moment frames can compensate to some degree for the loss of redundancy as well as enable the perimeter moment frame to act as a tube structural system (Youssef, Bonowitz and Gross, 1994). In recent years, most perimeter frames have been "partial" frames where only some spans of the perimeter frames have moment connections.

Another common type of steel moment frame system is the *planar* moment frame. Planar moment frames are included when mixed lateral load-resisting systems are used in a structure, generally, with moment frames in one direction and braced frames in another direction.

In recent years, particularly in southern California, moment frames with only a few spans, and sometimes only with one span have been used. In this system, selected spans in the entire planar frame have rigid connections while all other connections are shear connections. The columns that are not part of the moment frame are gravity columns and are not considered as part of the lateral load resisting system in design. Information on the actual behavior and design of the frames with only a few rigid spans was very limited and almost non-existent prior to the 1994 Northridge earthquake. Engelkirk (1994) provides some information on seismic design of steel moment frames with a few rigid bays. However, a large percentage of the steel structures damaged during the 1994 Northridge earthquake had this structural system.

The relatively poor performance of moment frames with only a few spans can be due to several factors. It appears that in these moment frames, the members and connections become extraordinarily large. As a result, it is possible that the large members (jumbo shapes) connected by very large size welds could not behave in a ductile manner due to material brittleness in large size welds and steel elements. Other factors such as characteristics of the ground motion and relatively small redundancy of the frames could have also contributed to the failures. The definite cause of these failures has been under investigation for the last five years and a number of parameters have been identified as possible contributors to fractures (SAC, 1994 and 1997).

Column-Tree Moment-Resisting Frames

In a column-tree moment-resisting frame system short segments of the girders, usually one to three feet long, are welded to the columns in the shop. Then, after the column-trees are erected in the field, the middle segment of the girders are bolted to the ends of the short girder stubs. Therefore, the system is a shop-welded, field-bolted steel structure. The shop welding provides high quality and economy as well as easy inspection. The field bolting results in the economy and ease of field erection and inspection as well as the possibility of year-round construction almost independent of weather conditions.

Various configurations of the rigid column-tree system have been used in the past in the United States. The shop-welded, field-bolted column-tree system is still popular for construction during cold weather as well as in the projects where welding is too costly or cannot be done easily. In Japan, perhaps due to the high cost of labor and the fact that shop welding is mostly automated, column-tree systems have been very popular. The performance of structures during the 1995 Great Hanshin Earthquake indicate that the modern engineered steel column tree systems in the affected areas performed well.

A semi-rigid version of the column-tree system was proposed by Astaneh-Asl (1988) in which the bolted connection of the girder, located away from the column, is made semi-rigid. Recently, a study of the rigid and semi-rigid column-tree systems was conducted at the Department of Civil Engineering of the University of California at Berkeley (McMullin et al, 1993). In the study, the semi-rigid column-tree system was shown to be a potentially reliable and economical seismic resisting structural system. The bolted semi-rigid connections at the location of girder splices act as fuses and protect the welded connections at the face of the columns from being subjected to large moments. In addition, the use of semi-rigid connections can increase damping, elongate period of vibration, reduce and control stiffness, and if done properly, result in reduction of seismic forces and displacements. More information on behavior and design of steel bolted column-tree moment frames can be found in Astaneh-Asl (1997).

Moment-Resisting Frames with Truss Girders

Moment frames with truss girders usually consist of rolled columns and welded steel truss girders. The information on seismic behavior and ductility of moment frames with truss girders is limited. An experimental and analytical study (Basha and Goel, 1994) provides information on the seismic behavior and design of a special ductile version of moment frames with truss girders. In this innovative system, the diagonal members of a few panels at midspan of the truss girders are removed. Testing and analysis of the resulting system has indicated good seismic behavior and potential for economical application in highly seismic areas.

Another version of steel moment frames with truss girders is a system where Vierendeel trusses are used as horizontal members. Recently, a seismic study of an existing 6-story structure, which has Vierendeel truss girders and is located near the Hayward fault in northern California, was conducted (Tipping, 1995). The inelastic time history analyses showed very good seismic behavior and well distributed yielding of the members of the truss girders.

Types of Moment Frames Based on Connection Method

These categories are based on how flanges of the girders are connected to the columns. The categories are

a) field welded;
b) field bolted;
c) riveted (were used until mid 1950s in the field and until 1960s in the shops).

Seismic behavior and design of welded moment connections have been studied since the 1970s. Due to damage in this type of connection caused by recent earthquakes, a number of studies have been undertaken both to understand the behavior of welded connections designed prior to the 1994 Northridge earthquake and to develop solutions to prevent welded connection damage. The reader can find extensive information on seismic behavior and design of welded moment frame connections in recent publications by the SAC-Joint Venture (SAC, 1994 and 1997) and other sources such as post-Northridge issues of *Modern Steel Construction* (AISC, 1994-present).

Examples of field bolted moment-resisting frame connections are shown in Figure 1. Bolted moment frames are defined as frames where no field welding is used and bolting is done in the field. In bolted, as well as welded moment frame connections, the transfer of shear force in the web of the girder is usually accomplished by welded and bolted shear connections.

Seismic behavior of bolted moment frame connections, shown in Figure 1, has also been studied by a number of researchers. A state-of-the-art paper by Leon in SAC (1997) provides a valuable summary of studies of bolted moment-resisting frames. Seismic behavior of bolted top-and-bottom plate moment connections have been studied by Harriott and Astaneh-Asl, (1990) and seismic design procedures and ductile detailing have been developed and presented (Astaneh-Asl, 1995). Seismic behavior of bolted rigid and semi-rigid top-and-bottom angle connections also has been studied using shaking table tests (Nader and Astaneh-Asl, 1992). A summary of these studies is provided later in this paper.

Recently, an experimental study of cyclic behavior of top-and-bottom stiffened angle connections were reported by Kasai et al. (1998). It appears that some particular details of the connections developed in this study are proprietary and not in the public domain.

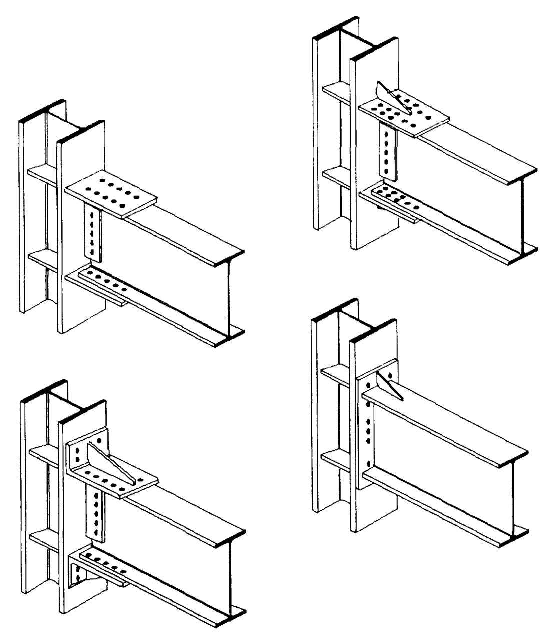

Fig. 1. Typical Field Bolted Steel Moment Frame Connections.

Categories of Moment Frames Based on Their Ductility

Steel moment-resisting frames are divided into two categories on the basis of their ductility. These are

a) special ductile moment-resisting frames;
b) intermediate moment-resisting frames;
c) ordinary moment-resisting frames.

It is well known that, depending on the extent of inelasticity in a structure, the magnitude of the seismic forces developed will vary. Inelasticity in steel structures can be due to yielding of steel, friction slip of bolts or limited inelastic buckling. Inelasticity usually reduces stiffness, causes energy dissipation, increases damping and elongates the periods of vibration. These changes in most common structures result in a reduction in the seismic forces and displacements developed. The current seismic design approach and code procedures are based on the concept of using inelasticity to reduce the seismic design forces. Steel moment frames are divided into Special, Intermediate and Ordinary on the basis of the source of inelasticity and the ability of the inelastic elements to deform while maintaining their strength. A brief summary of information on three types of steel moment frames is provided in the following. For more information the reader is referred to AISC Seismic Provisions (1997).

Special Moment-Resisting Frames

The connections and members of special moment-resisting frames are designed such that fracture and premature buckling of the structural members and connections are prevented. As a result, the special moment-resisting frames behave in a ductile manner. In special moment-resisting frames, the damage should be in the form of slippage, yielding of steel, delayed, and limited local buckling within the girder connections or within the girder plastic hinges. Fracture in any part that can impair the gravity-load carrying system should be avoided. This type of behavior categorizes the system as ductile.

Currently, there is debate in the profession on how much ductility supply is necessary for a given steel MRF to be categorized as a special ductile moment-resisting frame. Prior to the 1994 Northridge earthquake, some researchers (Popov, Ksai, and Englehardt, 1993) had suggested values of 0.015 and 0.02 radian to be the desirable rotation capacity of moment connections. However, the Northridge damage has cast serious doubt on these limits. On the basis of studies of rigid and semi-rigid moment-resisting frames, Nader and Astaneh-Asl (1992) had suggested a rotational ductility of 0.03 radian for moment connections in a special ductile moment resisting frame. A recent guideline released by the SAC-Joint Venture (SAC, 1994) as well as the AISC Seismic Provisions (AISC, 1997) specifies the inelastic rotation capacity of the special moment frames to be at least 0.03 radians.

In addition to the 0.03 radian rotation capacity, Astaneh-Asl (1995) suggested that the cumulative inelastic cyclic rotation capacity of a ductile moment connection to be at least 0.15 radian. This limit was suggested to ensure that after the connection reaches 0.03 radian rotation, it is not deteriorating rapidly and can sustain accumulated cyclic rotation of 0.15 radian to survive major quakes.

According to current seismic design codes (ICBO, 1997), the reduction factor for seismic design of steel special moment-resisting frames is 8.5 when used with load and resistance factor design methods.

Intermediate Moment-Resisting Frames

Compared to special moment-resisting frames, intermediate moment resisting frames have less stringent rotational ductility requirements. According to the AISC Seismic Provisions (AISC, 1997), the moment connections in intermediate moment-resisting frames should be able to demonstrate an inelastic rotation of at least 0.02 radians. Current codes do not specify a minimum cumulative rotational ductility. It is suggested herein that the cumulative cyclic rotational capacity for these connections be at least 0.10 radian.

Ordinary Moment-Resisting Frames

The steel ordinary moment-resisting frames need to have sufficient rotational ductility but not as much as intermediate and special moment frames. The AISC Seismic Provisions (AISC, 1997) indicates that the ordinary moment-resisting frames should demonstrate an inelastic rotation of at least 0.01 radians. The cumulative cyclic rotational capacity was suggested to be at least 0.10 radian.

Categories of Moment-Resisting Frames Based on Relative Stiffness and Strength of Girder and Connections

The behavior of a steel MRF strongly depends on the rotational stiffness and bending strength of its connections, girders and columns. Traditionally, steel moment-resisting frames are divided into three categories; Rigid (Fully Restrained, FR), Semi-rigid (Partially Restrained, PR) and Flexible (Simple) (AISC, 1994). Flexible moment frames can be found in some existing structures or are used as back-up systems for braced frame structures. The above division is primarily based on the rotational stiffness and bending strength of the beam-to-column joints. The parameters that have been used in the past to separate regions of rigid, semi-rigid and flexible are m and α defined as:

$$m = \frac{K_{con}}{\left(\frac{EI}{L}\right)_g} \qquad (1)$$

and

$$\alpha = \frac{(M_p)_{con}}{(M_p)_g} \qquad (2)$$

For definition of terms see Appendix-Notation of this paper.

Depending on rotational stiffness and moment capacity of the connection relative to the girder, connections are divided into rigid, semi-rigid and flexible (shear) connections. A number of criteria to define these three regions have been proposed in recent years. Nader and Astaneh-Asl (1992) developed the criteria that are given in the following, by modifying the traditional strength-based definition of the three types of connections. Considering the rotational stiffness and moment capacity of connections relative to girders, the definitions of rigid, semi-rigid and flexible connections are given as follows (Nader and Astaneh-Asl, 1992).

Rigid:		$m \geq 18.0$	and	$\alpha \geq 1.0$	(3)
Semi-rigid:	either	[$m > 18$	and	$0.2 < \alpha < 1.0$]	(4)
	or	[$18.0 \geq m \geq 0.5$	and	$\alpha > 0.2$]	
Flexible:	either	$m \leq 0.5$			(5)
	or	$\alpha < 0.2$			

Categories Based on the Moment Capacity of the Connected Members

Depending on relative bending capacities of columns and girders in the joints of a moment-resisting frame, the frame is categorized as one of the following:

a) Strong column-weak beam
b) Strong beam-weak column

In the strong column-weak beam frame, the moment capacity of the beams in a joint is less than the moment capacity of the columns. Therefore under combinations of gravity and lateral loads, plastic hinges are expected to form in the beams. In the strong beam-weak column design, plastic hinges are expected to form in the columns.

The strong column-weak beam frames are used very frequently in moment-resisting frames and many structural engineers believe that these systems have superior seismic behavior to that of the weak column-strong beam frames. In some frames, especially low rise structures, due to the long span of the girders or the heavy load on them, it is not easy nor economical to enforce the strong column, weak beam concept. However, by using semi-rigid connections, the capacity of end connections of girders can be adjusted such that the plastic hinges form in the connections and not in the columns.

SEISMIC BEHAVIOR OF BOLTED MOMENT-RESISTING FRAMES AND THEIR CONNECTIONS

Actual seismic behavior of structures can be studied by (a) investigation of the damage due to earthquakes and (b) by realistic laboratory testing of the structures and their components. Seismic performance of bolted steel moment frames during past earthquakes is briefly summarized herein followed by a brief summary of research projects and laboratory studies of behavior of bolted steel moment frames and their components.

Past Performance of Bolted Steel Moment-Resisting Frames

There are many existing riveted, bolted and welded steel structures that have been shaken by earthquakes in the past. No report of significant and consequential damage or collapse of major riveted moment-resisting frames could be found in the literature. One of the early tests of seismic performance of riveted steel structures was the 1906 San Francisco earthquake. In the post earthquake reports and photographs taken in the aftermath of the 1906 quake (Saul and Denevi, 1981), it appears that there was no collapse or structural damage to riveted steel structures in downtown San Francisco. All tall buildings of the time (all riveted steel structures) appear in photographs and reports to be undamaged. Alas, the later photographs, taken only a few days after the quake, show a few of the same buildings engulfed by the fire that swept through most of downtown San Francisco after the quake. In the photographs taken after the fire in San Francisco, there are several instances of steel column buckling and structural failures that appear to have been due to the intense heat of the fire reducing the strength of the members below their service load level, thus causing partial or total collapse of a number of steel structures. Of course today with the higher fireproofing standards and practices applicable to steel structures, such fire hazards are reasonably mitigated.

It is interesting to note that in the aftermath of the 1906 earthquake, the California State Board of Trade stated:

> "The earthquake damage was inconsiderable. Every building on both side of Market Street stood against the earthquake. The modern steel-frame buildings were unhurt, and that style of structure stands vindicated. The city has to rise from the ashes of conflagration, and not from the ruins of an earthquake." (California State Board of Trade, 1906).

Since the 1906 earthquake, there has been no published report of serious structural damage to bolted steel moment-resisting frames during earthquakes. Studies of performance of steel structures during the 1985 Mexico earthquake (Astaneh-Asl, 1986), (Martinez-Romero, 1988), the 1994 Northridge earthquake and the 1995 Kobe-Japan earthquake

(AIJ, 1995), (EQE, 1995), also indicate very good performance of bolted steel structures. It should be emphasized that most of the existing riveted and bolted moment-resisting frames were not designed and detailed as special ductile frames and can be categorized as ordinary moment-resisting frames. Therefore it is expected that some of them will experience damage during future major earthquakes. However, higher quality control for bolted steel structures relative to field-welded structures results in more redundancy in bolted connections, and less three-dimensional stress concentration effects. Thus, the likelihood of brittle damage in bolted connections appears to be less than the current field-welded connections.

In addition, because of slippage of the bolts and gap opening and closing in the connections, bolted steel structures demonstrate a limited amount of semi-rigidity during earthquakes. The author believes that the main reason for the good performance of bolted steel structures during past earthquakes is the semi-rigidity of bolted connections. In many cases, such semi-rigidity increases damping, releases and reduces stiffness to avoid large energy input, dissipates seismic energy, isolates the mass from the ground motions and elongates the period. All of these cause reduction in the seismic response of the structure. More information on performance and seismic design of steel semi-rigid moment frames can be found in Astaneh-Asl (1995) and Nader and Astaneh-Asl (1989 and 1992).

Behavior of Bolted Steel Moment-Resisting Frame Connections in Laboratory Tests

The systematic study of the cyclic behavior of steel moment connections started in the 1950s with the pioneering work of Egor Popov at the University of California, Berkeley and Ben Kato of the University of Tokyo. Since then a number of important research projects have been conducted in this field worldwide. The following sections provide a summary of selected projects that directly relate to the subject of this paper. For more information, the reader is referred to a state-of-the-art paper by Leon in SAC (1997).

Tests by Popov et al.

From the late 1950s through the late 1980s a series of cyclic tests and studies of the cyclic behavior of steel welded moment connections were conducted at the University of California at Berkeley (Pinkney and Popov, 1967), (Popov and Bertero, 1973) and (Popov and Stephen, 1972). The majority of connections tested were welded specimens with the exception of one project where bolted top- and bottom-plate connection specimens were also cyclically tested and studied. The performance of bolted specimens (Pinkney and Popov, 1967) is summarized here.

The specimens in the above tests consisted of a cantilever beam connected to a supporting column by top and bottom bolted plate connections. The specimens were subjected to cyclic moment by applying a cyclic load to the end of the cantilever beam. The failure modes observed in these specimens were local buckling of the beam and fracture of the net area of the beam or plate. In general, the top and bottom plates were stronger than the girder flange forcing the failure mode, in most cases, to be fracture of the net area of the girder flange. As the tests presented in the next section indicate, by following the current design procedures in the AISC Manual (AISC, 1994) for top and bottom plate connections, a more balanced and ductile design results. Such a balanced design results in the strengths of the connection and member being close and the damage being spread throughout the connection rather than concentrated along the net section of the girder flange.

Cyclic Tests of Bolted Top-and-Bottom Plate Moment Connections

Harriott and Astaneh-Asl (1990) conducted experimental and analytical studies of the cyclic behavior of bolted top-and-bottom plate moment connections. The objective was to investigate the cyclic behavior of three types of steel bolted beam-to-column connections under severe seismic loads. By using the information collected during the experiments, seismic design procedures for these connections were developed and proposed (Nader and Astaneh-Asl, 1992 and Astaneh-Asl, 1995). A refined version of these procedures is proposed later in this paper. Sketches of the beam-to-column connections that were tested are shown in Figure 2. Each specimen consisted of a seven foot long W18×50 beam connected to a three foot long column by top and bottom bolted flange plates and a shear connection. In all specimens the top and bottom plates were the same and were welded to the column by full

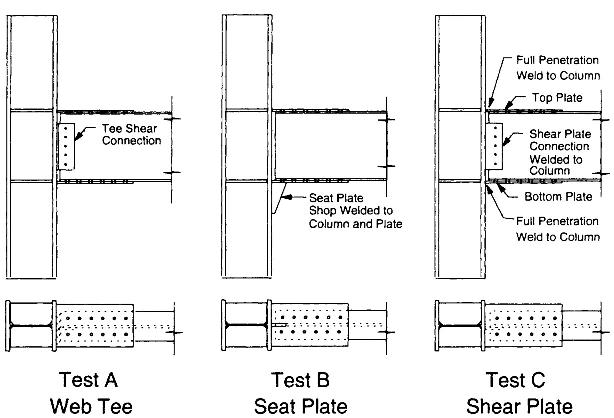

Fig. 2. Test Specimens for Bolted Top and Bottom-Plate Connections (Harriott and Astaneh-Asl, 1990).

penetration welds. The only difference among the specimens was the mechanism of shear transfer.

In Specimen A, the web connection was a structural tee. Specimen B did not have a web connection. To transfer shear from beam to column, in this specimen, a vertical stiffener was used under the bottom flange. The stiffener was welded to the column flange as well as to the bottom flange plate of the girder. Specimen C had a single-plate shear connection. The shear plate was welded to the column flange and bolted to the beam web using high strength bolts.

Figure 3 shows typical failure modes of welded and bolted rigid moment connections while Figure 4 shows a comparison of the moment-rotation behavior of a bolted connection (Harriott and Astaneh-Asl, 1990) and a comparable fully welded connection from the tests conducted by Popov and Bertero (1973).

The following observations are based on the results of the cyclic tests of bolted and welded connections reported in Nader and Astaneh-Asl (1989 and 1992) which were summarized above.

1. The initial elastic stiffnesses of bolted and welded specimens are almost the same. After several cycles of slip, the elastic stiffness of the bolted specimen is slightly less than that of the comparable welded specimen.
2. As cyclic loading continued, both the welded and bolted specimens continued to develop larger moment capacity albeit at much larger rotation.
3. The slip behavior of the bolted connections was very stable. The slope of the slip plateau was considerable indicating gradual slippage. At the end of the slip plateau, the bolted specimens were able to recover almost all of their initial elastic stiffness.
4. Because of slip and ductile yielding of the top- and bottom-plates and the shear connections, rotational ductility of the bolted specimens was nearly twice as much as that of comparable welded specimens.
5. In bolted specimens, there was almost no local buckling. Only very minor buckling was observed after at least ten inelastic cycles. In welded specimens, severe local buckling has been observed. In many cases, in welded specimens, the severity of local buckling was such that the locally buckled girder would need to be replaced after the earthquake in a real building.
6. In bolted specimens when a flange plate is subjected to compression, it yields in the area between the column weld and the first row of bolts. The same plate subjected to tension yields between the first and second rows of the bolt. In this case, the yield lines form along two lines making 45-degree angles with the line of application of the tension. In fully welded connections, both tension and compression yielding occur in the heat-affected zone of the welded flange adjacent to the weld line connecting the flange to the column as shown in Figure 3.
7. The cyclic behavior of the above bolted specimens was very ductile. All specimens could tolerate more than 15 inelastic cycles being able to reach cyclic rotations exceeding 0.035 radian.
8. As expected, the rotational stiffness of the connections was less than that predicted by the theoretical assumption of infinite rigidity. The elastic stiffness of the specimen with the web shear tab was almost the same as that of welded specimens tested by Popov and Bertero (1973) while the stiffness of specimens with the web tee connection and the seat connection was slightly less than that for the welded connections.
9. Slip in the bolted connections was small (about 1/8-in. after ten inelastic cycles).
10. In bolted connections, bending moment causing slip could be adequately predicted using a coefficient of friction of 0.33 given in the literature for unpainted clean mill scale (Class A) surfaces.

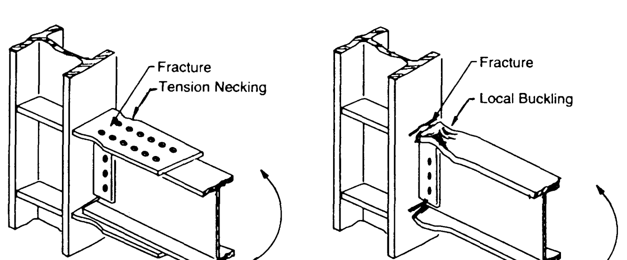

Fig. 3. Typical Failure Modes of Welded and Bolted Moment Connections.

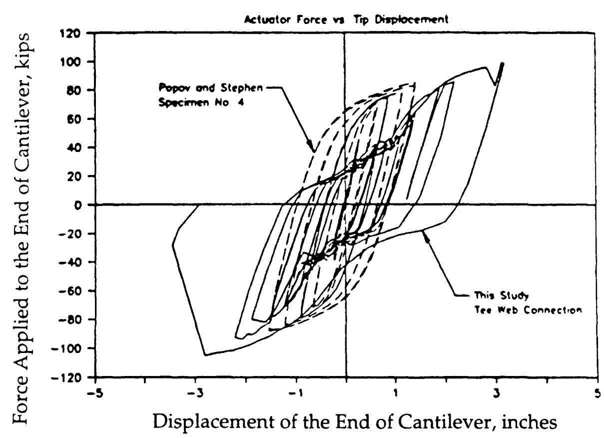

Fig. 4. Comparison of Moment Rotation Curves for Bolted and Welded Connections (Astaneh-Asl, 1995).

Finally, It should be added that the semi-rigidity observed in the bolted specimens does not necessarily reflect inferior characteristics for the seismic behavior of frames using these connections. As shown in the following section, shaking table tests (Nader and Astaneh-Asl, 1989) as well as analytical studies (Nader and Astaneh-Asl, 1992) have demonstrated that the semi-rigidity of ductile steel connections can improve the seismic response of steel frames.

Shaking Table Tests of Rigid, Semi-Rigid and Flexible Frames

In 1988 a series of 51 shaking table tests were conducted to study the behavior of welded and bolted, rigid, semi-rigid and flexible (simple) steel frames (Nader and Astaneh-Asl, 1989 and 1992). A one-story one-bay steel moment-resisting frame structure was constructed such that the beam-to-column connections could be replaced. Three types of connections, flexible, semi-rigid and rigid, were used resulting in flexible, semi-rigid and rigid frames.

The structure with three types of connections, one type at a time, was subjected to various levels of ground motions simulating 1940-El Centro, 1952-Taft and 1987-Mexico City earthquake records. A total of 51 shaking-table tests were conducted. The results of one series of tests, when rigid, semi-rigid and flexible structures were subjected to the Taft earthquake with maximum peak acceleration of 0.35g are summarized and discussed here. More information on the shaking table tests can be found in the report (Nader and Astaneh-Asl, 1989).

The shaking table tests indicated that the behavior of semi-rigid frame was quite good and in most cases better than the behavior of the similar but rigid frames. Drift of the semi-rigid bolted frame was only about 10 percent more than the drift of the rigid welded frame. The rigid frame behaved almost elastically when subjected to base acceleration time histories with maximum peak accelerations of 0.35g. The semi-rigid frame behaved in a very ductile manner and developed smaller base shear than the rigid frame but had slightly larger displacement. The behavior of the flexible frame was also stable and ductile with no traceable P-Δ effects. Figure 5 shows typical moment-rotation behavior of rigid and semi-rigid connections of the tested frames.

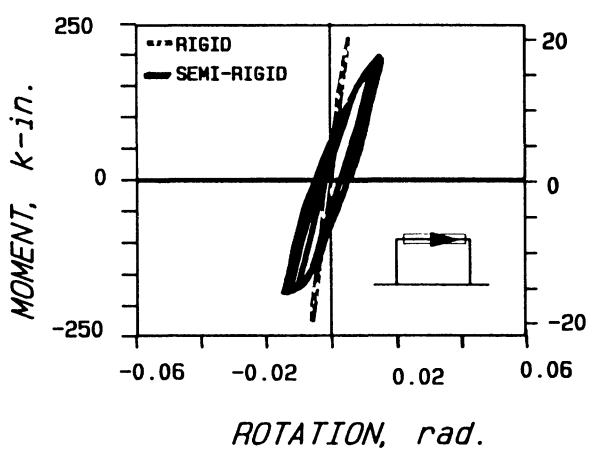

Fig. 5. Moment versus Rotation Curves for Connections (Nader and Astaneh-Asl, 1992).

SEISMIC ANALYSIS AND DESIGN OF BOLTED STEEL MOMENT-RESISTING FRAMES

Seismic design of bolted moment-resisting frames is similar to seismic design of welded moment-resisting frames. First, seismic lateral loads need to be established following the governing code. Second, seismic forces in combination with gravity loads are applied to a realistic model of the structure, and by analyzing the structure, component forces and nodal displacements are calculated. Finally, the components (members) and connections are designed to ensure that they have sufficient strength, stiffness and ductility for the applied forces and that the displacements of the structure do not exceed permissible limits.

One difference between welded and bolted moment connections is that in bolted moment connections, depending on the connection details, bolt slip and gap opening can occur during cyclic loading. Such usually small displacements are not expected to change the seismic behavior of rigid moment connections in an adverse manner. In fact, the available data indicates that such minor movements and release of stiffness in the connections can be beneficial in improving overall seismic behavior.

Another issue in seismic design of bolted moment frames is their service performance. To satisfy serviceability requirements, it is suggested herein that the seismic design of bolted connections be such that the slip of the bolts does not occur under service loading, but occurs prior to reaching the yield capacity of the connections. More discussion on this issue is provided in the Connection Design Philosophy in Special Moment-Resisting Frames section of this paper.

Issues Related to Computer Modeling of Bolted Moment Frames

A computer model of a bolted moment frame is built in the same manner as for other structures. One issue that needs special attention is bolt slip. Bolt slip within the connection, or deformations of the connection elements (angles and plates) can cause rotational stiffness of the connection to be less than that of a comparable welded connection. If the structural engineer desires to incorporate this flexibility into the analysis, it can be done by simply modeling the bolted connections as rotational springs in the computer model.

When a rotational spring is used to represent a bolted moment connection, the moment-rotation behavior of the connection should be established and modeled. Moment-rotation behavior can be established using available test results and a number of models proposed in the literature. However, using such models in a design office, currently poses some difficulties. Due to the large number of connections in a building frame, it is not easy or economical in a design office environment to establish sophisticated models for each connection. Also, most of the proposed models are empirical and developed by applying curve-fitting techniques to test results. It is not clear if such empirical models can be extrapolated directly and applied to other connections with different geometry and especially different material properties than those of the tested specimens.

One of the more practical ways of developing moment-rotation models of connections for design office use is by applying basic mechanics to model behavior of steel components and equilibrium of free bodies. Considering the uncertainties involved in the design process, material properties and loading; in most design applications, this approach based on fundamentals of mechanics and equilibrium should suffice. However, if more precise values of rotational stiffness of a particular bolted connection are needed, laboratory tests or finite element analysis methods can be used to establish such values. Of course such an undertaking will be far more expensive than the previously described mechanics-based approach and probably cannot be done economically and in due time in a design office environment where hundreds of connections in a building are to be designed. The use of generalized (non-dimensional) models of moment-rotation curves, such as those given by Equation 1 through 5, can be very useful in design offices. One of the benefits of non-dimensional design equations is that they are the same throughout the world regardless of the local unit system used. With design and construction becoming a global activity for many design firms, having globally applicable equations can facilitate checking the design and common intuitive feeling about the results.

Nader and Astaneh-Asl (1992) proposed equations based on mechanics of behavior and equilibrium that can be used to establish rotational stiffness of a number of common bolted moment connections. As an example of establishing rotational stiffness of a bolted moment connection, consider the top- and bottom-plate bolted moment connection. The moment rotation relationship for the connection is:

$$M_c = k_c \theta_c \tag{6}$$

The term k_c, for serviceability analysis, is the initial rotational stiffness, k_i, of the connection as shown in Figure 6. This stiffness represents, rotational stiffness of the connection before the bolts slip or significant inelastic deformation of connection elements take place. For ultimate strength design, k_c can be assumed to be the secant stiffness, k_s, of the connection, Figure 6. The secant stiffness includes flexibility of the bolted connection due to slip of the bolts and deformation of the connection elements.

To establish initial and secant stiffness for bolted top-and-bottom plate connections, Equation 6 can be rearranged and written in terms of axial displacement of the flanges:

$$k_c = \frac{M_c}{\theta_c} = \frac{F_f h}{\Delta_f / (h/2)} = \frac{F_f h^2}{2\Delta_f} \tag{7}$$

In the above equation, the term F_f / Δ_f is the axial stiffness felt by the girder flanges. The axial stiffness of the flange is provided by the flange top and bottom plates and friction slip between the girder flange and connection plates. Using Equations 6 and 7, the initial elastic rotational stiffness of the connection can be established as:

$$k_i = \frac{EA_p h^2}{L_p} \tag{8}$$

Assuming ⅛-inch slip for the bolts, the secant stiffness can be approximately given as:

$$k_s = \frac{EA_p h^2}{L_p}\left(\frac{1}{1 + \dfrac{2(1/8)}{(F_y/E)L_p}}\right) \tag{9}$$

It should be mentioned that the secant stiffness given by Equation 9 could be considered the elastic rotational stiffness of the top-and-bottom plate connections with *bolt slip included.* Such a reduced elastic stiffness of a connection can be used in design offices to estimate more realistic drift values under ultimate loads. If it is desired that drift values under service loads be estimated, the initial stiffness given by Equation 8 should be used.

Connection Design Philosophy in Special Moment-Resisting Frames

According to the latest edition of the Uniform Building Code (ICBO, 1997) and AISC Seismic Provisions (AISC, 1997) for special moment resisting frames, girder-to-column connections should be designed to develop at least the bending

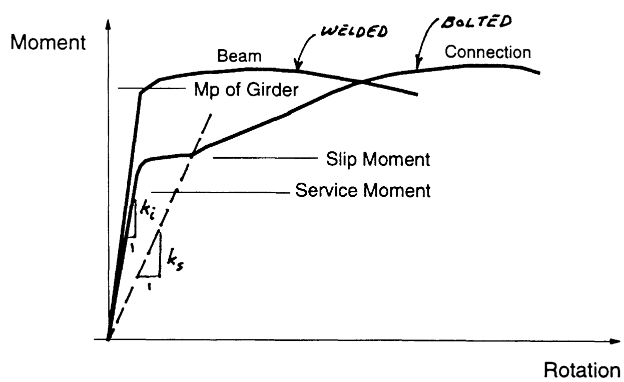

Fig. 6. Typical Moment-rotation Behavior of a Welded and Bolted Connection (Astaneh-Asl, 1995).

strength of the connected members, or to have sufficient ductility if it can be shown by laboratory tests.

To satisfy the general equation of design: *Capacity* $\geq$ *Demand*, the rotational ductility capacity of a moment connection should be greater than the rotational demand. Traditionally, ductility capacity of a steel moment-resisting connection is measured by cyclic moment rotation tests. As discussed earlier, the current consensus of the profession for rotational ductility demand of a special moment-resisting connection is 0.03 radians (SAC, 1994 and 1997). In designing special ductile moment-resisting frames, in the absence of a well-defined, reliable and universally accepted criterion to establish ductility demand, one rational approach is to focus on increasing the ductility supply of the connection. With the significant uncertainties that currently exist with regard to the characteristics of future earthquakes and their effects on the structure, the increased supply of ductility, above and beyond any specified demand (such as 0.03 radian) can improve the seismic performance of the structure significantly.

To increase supply of ductility, *ductile failure modes,* such as limited friction slip, yielding of steel and minor local buckling should be made the governing failure modes. The occurrence of brittle failure modes, such as fracture of welds and bolts or fracture of the net sections should be delayed and if possible prevented altogether.

In order to encourage formation of ductile failure modes prior to brittle failure modes, all possible failure modes of the connection should be identified. The failure modes should then be listed in the order of their desirability. Finally, performance-based design equations should be developed for each failure mode such that the capacities of ductile failure modes are less than the capacity of the brittle modes. This performance-based seismic design of connections based on a hierarchical order of failure modes can result in ductile and desirable behavior. The concept was used in developing design procedures currently in the AISC Manual (AISC, 1994) for design of shear tabs. In the following section, the concept is applied to seismic design of steel bolted top-and-bottom moment connections of special moment-resisting frames. Similar procedures were also developed by the author for other connections such as gusset plates (Astaneh-Asl, 1999).

Proposed Design Procedures for Bolted Top-and-Bottom Connections In Special Ductile Moment-Resisting Frames

In design of moment connections in seismic areas, three issues need to be addressed:

a) Stiffness
b) Strength
c) Cyclic ductility

In the following sections, these three issues, as related to design of bolted top-and-bottom plate moment connections are discussed.

Stiffness of Bolted Moment Connections (Stiffness Requirement)

The initial rotational stiffness of the connection relative to the girder should be large enough so that *the girder span* is categorized as rigid. This requirement is satisfied if m as defined in Equation (1) is greater than 18. In calculating m, the initial elastic stiffness, k_i, of the connection should be used.

Moment Capacity of a Bolted Moment Connection (Strength Requirement)

Moment capacity of a bolted moment frame connection is developed through strength of flange and web connections. Traditionally, in design, it is assumed that the flange connections provide the moment capacity and the web connections are designed to carry the shear force in the connection. This traditional approach has been proven to be a viable philosophy for design. The connections designed using this philosophy have performed well. Therefore, it is suggested herein that in seismic design of bolted moment connections this philosophy be followed. In reality, the shear connections of the web also participate in developing some moment capacity. But, such additional moment capacity is not considered in design of new connections. Perhaps in evaluating actual capacity of an existing moment connection, the contribution of web shear connections in resisting moment can be considered.

Design of flange connections according to the AISC Manual (AISC, 1994) assumes the applied moment is divided by the depth of the cross section, d. The connections of girder flanges are then designed for the force M/d. Following this method, flanges are designed to carry the entire applied moment without any help from the web connection. Again, as mentioned earlier, in reality the web and flange elements will share the load based on their stiffness and strength.

To ensure ductility of the connection, the governing failure modes of the flange connections should be ductile. Such ductile modes are friction slip, yielding of steel and *very* minor local buckling. Ductile modes usually do not need repair. On the other hand, failure modes such as fracture of welds or fracture of net areas should be avoided since these failure modes result in significant loss of strength and stiffness and in most cases may need expensive repairs.

To increase the ductility of connections in bending and to avoid costly damage to connections due to brittle fracture, it is suggested that first all possible failure modes of the connection are identified. Then, these failure modes are listed under a "hierarchy" reflecting their desirability. In this hierarchical list, more ductile failure modes are given higher standing and the less ductile failure modes such as fracture of bolts and welds are deemed undesirable and are given lower standing.

To ensure ductility of a steel connection, all failure modes should be identified and divided into two categories: ductile and brittle. Then the seismic design of the connection should be done such that the ductile failure modes govern the design. A suggestion to achieve this is to design for the capacity of the brittle failure modes to be 1.25 times the capacity of the largest ductile failure mode.

In order to develop seismic design procedures for bolted top-and-bottom plate connections, the following failure modes are identified and listed in hierarchical order of their desirability.

Ductile Failure Modes: When a component of a steel structure reaches a ductile limit state, the stiffness of the component is reduced significantly, but the strength of the component is more or less maintained. An example of a ductile limit state, or ductile failure mode, is yielding of steel. The following failure modes are considered ductile:

- Controlled and limited friction slippage of bolts
- Yielding of steel
- Minor local buckling that does not need repair

Brittle Failure Modes: When a component of a steel structure reaches a brittle limit state, both the stiffness and strength of the component are almost entirely lost. An example of a brittle limit state is fracture of the weld lines or bolt groups. The following failure modes are considered brittle:

- Severe local buckling that results in rapid deterioration of the material in a locally buckled area and leads to premature low-cycle fatigue fracture
- Fracture of bolt under shear, tension or combination of shear and tension
- Fracture of weld
- Fracture of steel section

Slip of the bolted component results in temporary loss of stiffness of the connection for a short duration during a seismic event. By designing the bolts to slip under a pre-determined level of force, the bolted connection can act as a fuse during earthquakes and limit the force that is transmitted through the bolts thus protecting the connection elements such as welds and bolts from fracture. In addition, friction results in significant local energy dissipation and damping. Because of the relatively large number of connections in bolted moment-resisting frames, slip can occur in many locations, in more or less random manner, dissipating the energy in a distributed and desirable manner.

Any bolted connection needs to slip and engage the bolts and connected steel parts before the bolts fail in shear. Therefore, slip of bolted connections subjected to shear is a natural phenomenon. The important question seems to be; when is the best time to let the bolts slip? Of course slippage of bolts under service loads can result in reduction of global stiffness and an increase in deformations. Such deformations cannot be acceptable under service load in many cases. On the other hand, if slippage occurs under a force level close to the shear failure capacity of the bolts, large amounts of seismic energy can be attracted to the structure and be stored in the form of strain energy. When slip finally occurs, the resulting impact and the fact that the slippage force is close to the fracture capacity can cause the bolts to fail in shear. To safeguard against such undesirable performance and to satisfy serviceability, the following criteria for bolt slip under seismic loads is suggested:

$$1.25F_{Service} \leq F_{Slip} \leq 0.80F_{Ultimate} \tag{10}$$

The 1.25 and 0.80 factors in the above equation are introduced to provide a reasonable margin of safety against slip under the service condition as well as to guard against slip occurring too close to the ultimate capacity. Unfortunately test results on cyclic slip behavior of steel structures are very limited. As a result, the reader is cautioned that the above limits of 1.25 and 0.8 are selected primarily based on engineering judgment and intuition, and are therefore, subject to the judgment and approval of the structural engineer in charge of the design.

Local buckling can be categorized as ductile or brittle depending on how rapidly the locally buckled area deteriorates during cyclic loading. Available cyclic test results (Astaneh-Asl, Goel and Hanson, 1985) indicate that steel members with high *b*/*t* ratios, say higher than $\lambda_r = 95/\sqrt{F_y}$ given in the AISC Specifications (AISC, 1994), tend to form severe local buckling in a very sharp configuration, develop relatively large out-of-plane deformations of locally buckled areas and fracture through the sharp tip of the locally buckled areas after a few inelastic cycles. Cyclic local buckling in this manner should be considered brittle. Members with a *b*/*t* ratio less than those specified by the AISC Seismic Provisions (AISC, 1997) tend to develop local buckling after a relatively large number of inelastic cyclic deformations (usually more than 10 to 15 cycles of inelastic behavior before local buckling). The limit for the *b*/*t* ratio for the flanges of the girders currently given in the AISC Seismic Provisions (AISC, 1997), is $52/\sqrt{F_y}$.

When the *b*/*t* ratio of the flange is less than $52/\sqrt{F_y}$, the locally buckled area does not develop a sharp tip. This type of local buckling can be considered sufficiently ductile.

For members with *b*/*t* ratios greater than $52/\sqrt{F_y}$ and less than $95/\sqrt{F_y}$ there is not sufficient data on their low-cycle fatigue behavior to be used to establish their cyclic ductility. In a conservative move and until more test data becomes available, cyclic local buckling of members with *b*/*t* ratios between $52/\sqrt{F_y}$ and $95/\sqrt{F_y}$ is suggested to be considered nonductile (brittle) in seismic Zones 3 and 4 and sufficiently ductile for seismic Zones 1 and 2. It is suggested that the *b*/*t* ratio of any member in a steel structure in any seismic zone be less than $95 / \sqrt{F_y}$ if there is a possibility of that member to be subjected to seismic loads.

The following guideline, which is based on the monotonic and cyclic local buckling behavior of steel members, is conservatively suggested by the author to be used to categorize local buckling failure modes as ductile or brittle in seismic Zones 3 and 4:

If $b/t < 0.80\lambda_p$, behavior is ductile, otherwise behavior is considered to be nonductile (brittle), where λ_p is the limit for the b/t ratio for plastic design of steel structures given in Table B5.1 of the AISC Specification (AISC, 1994). This table gives the value of λ_p for flanges of the rolled wide flange shape as $65 / \sqrt{F_y}$.

SEISMIC DESIGN PROCEDURES FOR BOLTED TOP AND BOTTOM FLANGE MOMENT CONNECTIONS

For a bolted top-and-bottom plate flange connection, the major failure modes are given in the following hierarchy with the first mode being the most ductile (desirable) and the last mode the most brittle (undesirable).

Hierarchy of Failure Modes for Bolted Top-and-Bottom Plate Moment Connections

Figure 7 shows failure modes of a bolted top-and-bottom plate moment connection and in a hierarchical way from left-to-right of the figure.

Ductile Failure Modes for Flange Connections:

a) Slip of the flange bolts
b) Yielding of the gross area of the girder flange
c) Yielding of the gross area of the top and bottom flange plates
d) Bearing yielding of the bolt holes in the girder flanges and the flange plates
e) Yielding of edge distance of bolts

Failure Modes with Limited Ductility for Flange Connections:

f) Local buckling of the top and bottom flange plates
g) Local buckling of the girder flanges
h) Shear yielding of the column panel zone

Brittle Failure Modes for Flange Connections:

i) Shear fracture of the flange bolts
j) Block shear failure
k) Fracture of the edge distance or bolt spacing
l) Fracture of the net section of the flange plate
m) Fracture of the welds connecting the top and bottom plates to the column
n) Net section fracture of the girder flanges

In the above list, failure modes (a) through (e) are considered ductile and desirable. Failure modes (f) and (g) are considered ductile provided that b/t ratios satisfy the limit given in the Moment Capacity of a Bolted Moment Connection (Strength Requirement) section above. The column panel zone yielding (h) is considered ductile if panel zone design satisfies the requirements of the Uniform Building Code (ICBO, 1997). Failure modes listed as (i) through (n) are considered brittle and not acceptable to govern the strength of the bolted special moment-resisting frames.

Slippage of Flange Bolts

Comprehensive information on the slip behavior of bolted connections has been given by Kulak et al. (1987) and in AISC (1994), Volume II. The important question for bolted special moment connections in seismic regions is that: should the bolted connections in special moment frames be permitted to slip, and if slip is permitted, at what level of load should it occur?

As discussed earlier, it is suggested that the bolted moment connections be designed such that slip does not occur at or below service load but occurs prior to reaching 80 percent of bolt shear capacity. This can be achieved by satisfying Equation 10.

Yielding of Gross Area of Girder

This failure mode occurs when a plastic hinge forms in the girder and should be the target failure mode in seismic design of rigid connections. As indicated throughout this section, other failure modes are matched against this desirable failure mode. The equation to establish plastic moment capacity of the girder is:

$$M_{p\,(girder)} = F_y Z \tag{11}$$

Yielding of Gross Area of Top and Bottom Plates

To increase ductility of the connection, yielding of top and bottom flange plates should be encouraged as the girder enters strain hardening. To achieve this, it is suggested that the plastic moment that causes plate yielding should be close to

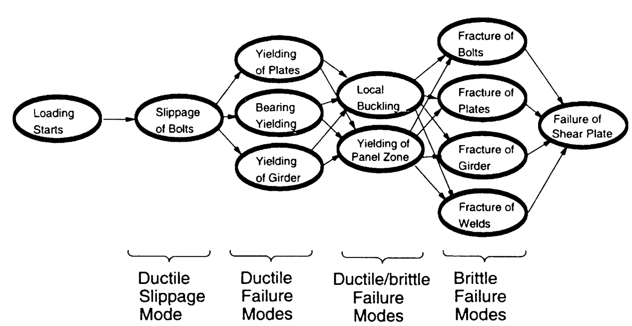

Fig. 7. Hierarchical Order of Failure Modes of Bolted Top-and-Bottom Flange Plate Moment Connections(Astaneh-Asl, 1995).

or slightly greater than 1.25 times the plastic moment capacity of the girder,

$$M_{p\,(plates)} \geq 1.25 M_{p\,(girder)} \tag{12}$$

Bearing Yielding of Bolt Holes in Girder Flange and Plates

Bearing yielding of the bolt holes is beneficial in reducing seismic response during extreme events. It is suggested that in design the moment that can cause bearing yielding in the connection is equal to or slightly greater than 1.25 times the yield moment of the girder, as expressed in:

$$M_{p\,(bearing)} \geq 1.25 M_{p\,(girder)} \tag{13}$$

Yielding Related to Edge Distance

When a bolt close to the edge of a plate is bearing against the plate, the portion of plate that is between the bolt hole and the edge of the plate is subjected to bending and shear. This portion of the plate, acting as a short beam can yield under the combination of shear and bending resulting in ovalization of the holes. This phenomenon has been observed in laboratory tests (Liu and Astaneh-Asl, 1999). If limited yielding of the plate edge and a small amount of ovalization occurs under loads exceeding service loads, it can be beneficial in increasing the ductility of the connection. It appears that in the absence of a systematic study of this item, and based on observations of cyclic behavior of top-and-bottom plate moment connections (Harriott and Astaneh-Asl, 1990) and shear tabs (Liu and Astaneh-Asl, 1999), the edge distance limitations provided in the AISC Specifications (AISC, 1994) should be adequate for a desirable behavior and should be followed.

Local Buckling of the Top and Bottom Flange Plates

Based on cyclic behavior of top-and-bottom plate moment connections, as discussed earlier, it appears that in these connections, the flanges of the girder and the plates brace each other to some extent delaying local buckling of the plate as well as the girder flange. The portion of the top and bottom flange plates between the first row of the bolts and the weld line is the most stressed region in compression and should be checked for buckling. This portion of a plate should be made as short as is practically possible. Considering clearances and the space needed around the bolts for tightening, the distance of the first row of bolts from the column face will be on the order of 4 to 5 times the diameter of the bolts in most practical situations. Longer spaces are not desirable since that can facilitate buckling of the plates during compression cycles and reduce the rotational rigidity of the connection. A shorter length for this portion can result in concentration of plasticity near or within the heat-affected zone resulting in premature fracture.

To prevent premature local buckling of top and bottom flange plates, it is suggested that the ratio of free length of the plate (the length between the last row of bolts and the column weld line) to thickness of the plate be less than 6. This results in a KL/r of about 25 for the plate buckling.

Local Buckling of Girder Flanges

As discussed earlier, if the b/t ratio of the girder flange is less than $52/\sqrt{F_y}$, local buckling of the girder flange will be sufficiently delayed during a cyclic event. When the cyclic local buckling occurs it will be relatively smooth and ductile without significant loss of strength.

Shear Yielding of the Panel Zone

To obtain a ductile behavior due to shear yielding of panel zone, the reader is referred to the current provisions of the Uniform Building Code (ICBO, 1997).

Shear Fracture of the Flange Bolts

After slip of the bolts and some bearing yielding, the applied moment is totally carried by shear in the bolts. To encourage yielding of steel before bolt shear failure, the following criterion is suggested:

$$\phi_b F_b A_b N d \geq 1.25 \phi M_{p\,(girder)} \tag{14}$$

Block Shear Failure

Block shear failure is a fracture-yield type of failure where the boundary of a block of steel plate yields in some areas and fractures in others. To ensure that this relatively brittle failure mode does not occur before the plates yield, the following criteria is suggested:

$$\phi_n P_n \geq 1.25 \phi M_{p\,(girder)}/d \tag{15}$$

(a) When $F_u A_{nt} \geq 0.6 F_u A_{nv}$

$$P_n = 0.6 F_y A_{gv} + F_u A_{nt} \tag{16}$$

(b) When $F_u A_{nt} < 0.6 F_u A_{nv}$

$$P_n = 0.6 F_u A_{nv} + F_y A_{gt} \tag{17}$$

Fracture Related to Edge Distance or Bolt Spacing

Fracture within the edge distance by itself may not be catastrophic, but during cyclic loading a crack within the edge distance can jump the bolt hole and fracture the entire width of the plate or flange of the girder. On the basis of the limited information currently available on the cyclic behavior of bolt edge distances, it is suggested that in special moment frames bolt edge distances be greater than 1.5 times the diameter of the bolt and preferably 2 times the diameter. In most bolted top and bottom connections, there is sufficient width of plate or girder flange to easily accommodate an edge distance equal to two bolt diameters.

The minimum bolt spacing is specified by the AISC Specifications (1994) to be 2.66 times the diameter of the bolts with a spacing of 3 times the bolt diameter preferred. In the United States it is common practice in fabrication shops to employ a bolt spacing of 3 inches for almost all bolts up to 1¼ inch in diameter. In the absence of any report of failure of bolt spacing during earthquakes or in laboratory tests, it appears that 3-inch spacing is satisfactory.

Fracture of Net Section of the Flange Plates

The plates should be designed such that the fracture of plates does not occur before yielding and strain hardening of the girder. The following criterion is suggested:

$$\phi_n M_{pn} \geq 1.25 \phi M_{p\,(girder)} \tag{18}$$

Fracture of the Welds

The welds connecting the top and bottom plates to the columns should be full penetration groove welds done in the shop. It is recommended that in design of these welds, the latest provisions of the AWS be followed.

Net Section Fracture of the Girder Flanges

If net sections of the flanges of the girder fracture, it is possible that the crack will propagate into the girder web. During or after a quake, the cracked web of the girder may not be able to carry the service gravity load and the crack could propagate across the entire section and result in the collapse of the span. Since such a scenario is not acceptable, fracture of the net section of the girder is considered very undesirable. The following criterion is suggested to check against such failures (Astaneh-Asl, 1995).

$$\frac{A_e}{A_g} \geq \frac{1.25 F_y}{F_u} \tag{19}$$

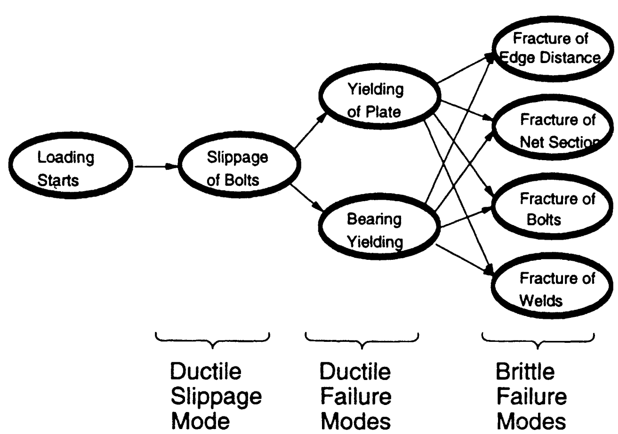

Fig. 8. Hierarchical Order of Failure Modes of Shear Tab Connections (Astaneh-Asl et al, 1988).

Design of Shear Connections

Design of shear connections in bolted moment frame connections are done following the AISC Specifications (AISC, 1994) and the information available in the literature. For behavior and design of shear tab connections the reader is referred to Astaneh-Asl, McMullin, and Call (1988), Astaneh-Asl, Call, and McMullin (1989), Astaneh-Asl (1989a and 1989b) and Liu and Astaneh-Asl (1999). Figure 8 shows the hierarchical order of failure modes for shear tab connections (Astaneh-Asl, 1989b) that was used in developing the current design procedures for shear tabs in the AISC Manual (AISC, 1994). Information on behavior and design of tee and double angle shear connections can be found in Astaneh-Asl and Nader (1989 and 1990) and Astaneh-Asl, Nader, and Malik (1989).

Of course one of the most important elements of bolted connections is the bolt itself. Due to space limitations, it was not possible here to discuss bolt behavior. The reader is referred to Kulak, Fisher and Struik (1987) for information on bolts.

CONCLUDING REMARKS

The main goal of this paper was to provide design-oriented information on seismic behavior and design of steel bolted moment connections. It was shown that based on seismic behavior of bolted moment frames and connections in the field as well as in the laboratory, these structures could be a viable and economical lateral load-resisting system.

The proposed performance-based seismic design procedures based on the concept of hierarchical order of failure modes, from ductile to brittle, can be used not only in design of bolted moment frame connections, but also in seismic design of any structural component.

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APPENDIX-NOTATION

A_b = area of one bolt
A_{pb} = gross area of one flange plate in the area between the first bolt line and the weld line.
A_{gv} = gross area subject to shear
A_{gt} = gross area subject to tension
A_{nv} = net area subject to shear
A_{nt} = net area subject to tension
A_{np} = net area of one plate across the first bolt row
d = back-to-back depth of girder
d_b = diameter of bolts
$F_{Ultimate}$ = shear capacity of one bolt
F_b = shear capacity of one bolt
F_f = flange force
F_{up} = minimum specified tensile strength of the plates
$F_{service}$ = applied service shear force
F_y = specified yield stress
F_{yield} = yield capacity of flange
$F_{slippage}$ = force that can cause friction slippage,
I = moment of inertia of girder
K_{conn} = rotational stiffness of connections
k_c = rotational stiffness of connections
k_i = initial stiffness of connections
k_s = secant stiffness of connections
L = span of the girder
L_p = length of flange plate from weld line to last bolt line
M_{pn} = plastic moment capacity of the net section of the plates $= F_u d A_{np}$
$M_{p\,(bearing)}$ = bearing moment capacity of the girder
$M_{p\,(girder)}$ = plastic moment capacity of the girder
$M_{p\,(plates)}$ = moment causing yielding of the top and bottom plates $= F_{yp} A_p d$
M_c = connection moment
m = stiffness parameter
N = number of bolts
P_n = nominal resistance of flange plate in block shear failure
t = thickness of the plate or flange
Z = plastic section modulus of the girder cross section
Δ_f = movement of flange of the girder relative to column face
ϕ = resistance reduction factor for yielding = 0.90
ϕ_n = resistance reduction factor for fracture =0.75
ϕ_b = resistance reduction factor for fracture = 0.75
λ_p = limit of b/t for plastic local buckling, (see AISC (1994))
λ_r = limit of b/t for elastic local buckling, (see AISC (1994))
θ_c = connection rotation